Appendix A
COMMENTARY ON THE SELECTION OF THE DESIGN EARTHQUAKES

A.1 INTRODUCTION

This appendix describes the design earthquakes and associated ground motions that have been adopted for the proposed revisions to the AASHTO *LRFD Bridge Seismic Design Specifications*.

For applicability to most bridges, the objective in selecting design earthquakes and developing the design provisions of the specifications is to (1) preserve live safety and prevent bridge collapse during rare earthquakes and (2) provide immediate (except for inspections) post-earthquake serviceability of bridges with minimal damage during expected earthquakes. For applicability to certain bridges of special importance as determined by the bridge owner, performance objectives may be higher than stated above.

The following sections of this appendix are organized as follows: Section A.2 provides a brief description of the design earthquake ground motion map in the current AASHTO LRFD Provisions. Sections A.3 and A.4 describe earthquake ground motion maps that are proposed for these revised LRFD Seismic Specifications. Section A.5 describes the proposed design earthquakes and associated ground motions utilizing the new ground motion maps. Finally, Section A.6 summarizes the results of studies conducted to evaluate the impacts of the proposed revised specifications on bridge construction costs. Additional discussion and analyses of earthquake ground motion maps, site factors, and response spectrum construction procedures may be found in publications by ATC/MCEER (1999a, 1999b).

A.2 CURRENT AASHTO MAP (1990 USGS MAP)

The national earthquake ground motion map in the current AASHTO LRFD Bridge Seismic Design Specifications is a probabilistic map of peak ground acceleration (PGA) on rock developed by the U.S. Geological Survey (USGS) (1990). The map provides contours of PGA for a probability of exceedance of 10% in 50 years. The PGA map is used with rules contained in the AASHTO Specifications for obtaining seismic response coefficients or response spectral accelerations.

A.3 NEW USGS MAPS

In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. In California, the mapping project was a joint effort between USGS and the California Division of Mines and Geology (CDMG). The result of that project was a set of probabilistic maps published in 1996 for the conterminous United States and subsequently for Alaska and Hawaii that cover several rock ground motion parameters and three different probability levels or return periods (Frankel et al., 1996, 1997a, 1997b, 1997c, 2000; Frankel and Leyendecker, 2000; Klein et al., 1999; Peterson et al., 1996; Wessen et al., 1999a, 1999b). The maps are available as large-scale paper maps, as small-scale paper maps obtained via the Internet, and as digitized values obtained from the Internet or a CD-ROM published by the USGS (Frankel and Leyendecker, 2000).

Parameters of rock ground motions that have been contour mapped by USGS include peak ground acceleration (PGA) and response spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 second. Contour maps for these parameters have been prepared for three different probabilities of exceedance (PE): 10%

PE in 50 years, 5% PE in 50 years, and 2% PE in 50 years (approximately equal to 3% PE in 75 years), corresponding, respectively, to approximate ground motion return periods of 500 years, 1000 years, and 2500 years. In addition to these contour maps, the ground motion values at locations specified by latitude and longitude can be obtained via the Internet for the aforementioned three probability levels for PGA and spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 seconds. The CD-ROM published by the USGS also provides spectral accelerations at additional periods of 0.1, 0.5, and 2.0 seconds. In addition, the CD-ROM contains not only the PGA and spectral acceleration values at three probability levels but also the complete hazard curves (i.e., relationships between the amplitude of a ground motion parameter and its annual frequency of exceedance [annual frequency of exceedance is the reciprocal of return period]) for specified latitudes and longitudes. Therefore, ground motion values can be obtained for any return period or probability of exceedance from the hazard curves on the CD-ROM.

The new USGS national ground motion mapping incorporated inputs for seismic source models and ground motion attenuation models that represent major improvements over the models used for the current AASHTO maps with regard to capturing the state of scientific knowledge. Some of the key areas of incorporation of updated scientific knowledge for the new USGS maps include:

1. Much more extensive inclusion of identified discrete active faults and geologic slip rate data. Approximately 500 faults were incorporated in the mapping. Geologic slip rates for these faults were utilized to determine earthquake recurrence rates for the faults.

2. Improved and updated seismicity catalogs were utilized in determining earthquake recurrence rates for seismic sources not identified as discrete faults. In the central and eastern United States (CEUS), these catalogs utilized updated assessments of magnitudes of pre-instrumental older earthquakes (originally characterized by their Modified Mercalli Intensity). These assessments had the effect of reducing the estimated rate of larger earthquakes in the CEUS (equal to or greater than approximately magnitude 5).

3. In the Pacific Northwest (Washington, Oregon, and northwest California), the Cascadia subduction zone seismic source was explicitly included. Geologic/paleoseismic data were utilized to characterize the recurrence rate of very large earthquakes (magnitude 8 to 9) occurring in the coastal and offshore regions of the Pacific Northwest.

4. Geologic/paleoseismic data were utilized to characterize the recurrence rate of large earthquakes occurring in the New Madrid seismic zone (in the vicinity of New Madrid, Missouri) and the Charleston seismic zone (in the vicinity of Charleston, South Carolina).

5. Updated, recently developed ground motion attenuation relationships were utilized. These relationships incorporated the developing knowledge of differences in ground motion attenuation relationships in different regions and tectonic environments of the United States. As a result, different attenuation relationships were used in the CEUS, shallow-crustal faulting regions of the WUS, and subduction zone regions of the Pacific Northwest and Alaska.

The new probabilistic maps developed by the USGS have been widely accepted as providing a greatly improved scientific portrayal of probabilistic ground motions in the United States compared to earlier maps. These maps were assessed for possible utilization for seismic design of bridges and other highway facilities by the 1997 FHWA/MCEER workshop on the National Characterization of Seismic Ground Motion for New and Existing Highway Facilities (Friedland et al., 1997). The workshop concluded that "…these new maps represent a major step forward in the characterization of national seismic ground motion. The maps are in substantially better agreement with current scientific understanding of seismic sources and ground motion attenuation throughout the United States than are the current AASHTO maps. …the new USGS maps should provide the basis for a new national seismic hazard portrayal for highway facilities…"

The USGS has in place a systematic process for periodically updating the maps to reflect continuing advances in knowledge of earthquake sources and ground motions. Therefore organizations using these maps (or maps adapted from the USGS maps as described below) have the opportunity to update the maps in their seismic criteria documents as appropriate.

A.4 NATIONAL EARTHQUAKE HAZARD REDUCTION PROGRAM (NEHRP) MAXIMUM CONSIDERED EARTHQUAKE (MCE) MAPS

The federal Building Seismic Safety Council (BSSC) adopted a modified version of the new USGS Maps for 2% PE in 50 years to define the recommended ground motion basis for the seismic design of buildings in the 1997 NEHRP Provisions (BSSC, 1998; Leyendecker et al., 2000a, 2000b). These maps are termed the Maximum Considered Earthquake (MCE) maps and are presented in this Specification in Figures 3.10.2.1-1(a) through 3.10.2.1-1(l). The maps are for 0.2-second and 1.0-second response spectral accelerations. Map values for locations specified by latitude and longitude may be obtained from a CD-ROM published by USGS (Leyendecker et al., 2000a).

The 1997 NEHRP MCE maps are identical to the new USGS maps for a probability of ground motion exceedance of 2% in 50 years (return period of approximately 2500 years), except that in areas close to highly active faults, "deterministic bounds" are placed on the ground motions with the intent that ground motions are limited to levels calculated assuming the occurrence of maximum magnitude earthquakes on the faults. The deterministic bounds are defined as 1.5 times the median ground motions calculated using appropriate ground motion attenuation relationships (the same relationships as used in the USGS probabilistic mapping) assuming the occurrence of maximum magnitude earthquakes on the faults, but not less than 1.5g for 0.2-second spectral acceleration and 0.6g for 1.0-second spectral acceleration. Multiplying the median ground motions by 1.5 results in ground motions that are approximately at a median-plus-standard-deviation level (actually somewhat lower, in general, because the ratio of median-plus-standard-deviation ground motions to median ground motions usually exceeds 1.5). Figure A-1 conceptually illustrates the procedure for incorporating deterministic bounds on the MCE maps. The deterministic bounds limit ground motions to values that are lower than those for 2% PE in 50 years in areas near highly active faults in California, western Nevada, coastal Oregon and Washington, and parts of Alaska and Hawaii.

A.5 DESIGN EARTHQUAKES

Two design earthquakes are defined for this specification. The upper level earthquake is the "rare" earthquake and is defined as the MCE described in the preceding section. For a bridge design life of 75 years, the ground motions for the MCE correspond to 3% PE in 75 years, except that lower ground motions are defined in areas of deterministic bounds as described above. The lower level earthquake is the "expected" earthquake and is defined as ground motions corresponding to 50% PE in 75 years.

A.5.1 RARE EARTHQUAKE (MCE)

The intent of the MCE is to reasonably capture the maximum earthquake potential and ground motions throughout the United States. As summarized in Section A.1, the design objective is to preserve life safety and prevent collapse of the bridge, although some bridges may suffer considerable damage and may need to be replaced following the MCE.

In the current AASHTO Specifications, a 10% probability of exceedance in 50 years, or approximately a 500-year return period, is used. However, based on a detailed analysis of the new USGS maps (ATC/MCEER, 1999a; 1999b), the ground motions over much of the United States increase substantially for probability levels lower than 10% in 50 years or return period longer than 500 years. The increase in ground motions with return period is illustrated in Figures A-2(a) and A-2(b). In these figures, ratios of 0.2-second and 1.0-second spectral accelerations for given return periods to 0.2-second and 1.0-second spectral accelerations for an approximate 500-year return period are plotted versus return period for selected cities in three regions of the conterminous United States: central and eastern United States (CEUS); western United States outside California (WUS); and California. In California and coastal Oregon and Washington, the effects of deterministic bounds described in Section A.4 on the ground motion ratios are included where applicable. The curves in Figures A-2(a) and A-2(b) illustrate that MCE ground motions in areas of deterministic bounds in highly seismically active areas of California do not greatly exceed 500-year ground motions, with ratios of MCE to 500-year ground motions typically in the range of about 1.2 to 1.5. However in other parts of the WUS and in the CEUS, ratios of MCE ground motions (i.e. approximately 2500-year ground motions except where deterministically bounded) to 500-year ground motions typically range from about 2 to 2.5 in the WUS and 2.5 to 3.5 in the CEUS. Even higher ratios are obtained for some areas exposed to large magnitude characteristic earthquakes having moderately long recurrence intervals defined by paleoseismic data, such as Charleston, New Madrid, Wasatch Front, and coastal Oregon and Washington. These results motivate the recommendation to adopt MCE ground motions as a design basis for a "no collapse" performance criterion for bridges during rare but credible earthquakes.

Analysis of 1996 USGS map ground motions in the Charleston, South Carolina and New Madrid, Missouri regions also indicate that 500-year return period ground motions within 75 km of the source region of the 1811-1812 New Madrid earthquakes and the 1886 Charleston earthquake are far below the ground motions that are likely to have occurred during these historic earthquakes. However, 2500-year return period ground motions are in much better agreement with ground motions estimated for these earthquakes. If deterministic estimates of ground motions are made for the historic New Madrid earthquake of estimated moment magnitude 8.0 using the same ground motion attenuation relationships used in the USGS probabilistic ground motion mapping, then the 500-year mapped ground motions are at or below the deterministic median-minus-standard-deviation ground motions estimated for the historic events within 75 km of the earthquake sources, whereas 2500-year ground motions range from less than median to less than median-plus-standard-deviation ground motions. Similarly, 500-year ground motions range from less than median-minus-standard-deviation to less than median ground motions deterministically estimated for the 1886 Charleston earthquake of estimated moment magnitude 7.3 within 75 km of the earthquake source; whereas 2500-year ground motions range from less than median to slightly above median-plus-standard-deviation ground motions for this event. It is desirable that design ground motions reasonably capture the ground motions estimated for historically occurring earthquakes.

Adoption of the MCE as a design earthquake for a collapse-prevention performance criteria is consistent with the adoption of the MCE in the 1997 and 2000 NEHRP Provisions for new buildings (BSSC, 1998; BSSC, 2001), the 2000 International Building Code (IBC) (ICC, 2000), and the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (ATC and BSSC, 1997). In the 1997 and 2000 NEHRP Revisions for new buildings and the 2000 IBC, the MCE ground motions are defined as collapse prevention motions but design is conducted for two-thirds of the MCE ground motions on the basis that the design provisions in those documents (including the R-Factors) would provide a minimum margin of safety of 1.5 against collapse. On the other hand, in the NEHRP Guidelines for Seismic Rehabilitation of Buildings, the MCE ground motions are proposed to be directly used as collapse prevention motions in design. The approach proposed for these specifications is similar to that of the NEHRP Seismic Rehabilitation Guidelines in that the design provisions for the MCE have been explicitly developed for a collapse-prevention performance criterion.

The decision to use the 3% PE in 75 year event with deterministic bounds rather than 2/3 of this event (as used in the 2000 NEHRP provisions) was to directly address and incorporate design displacements associated with the MCE event. Displacements are much more important in bridge design because they govern the seat width of girders supported by columns and thus are critically important in preventing collapse.

A.5.2 EXPECTED EARTHQUAKE

The intent of the expected earthquake is to describe ground motions that are expected to occur during a 75-year bridge life (with a 50% probability of being exceeded during the bridge life). Design is for minimal damage and normal service following post-earthquake inspection. Expected earthquake ground motions are defined by the new USGS probabilistic ground motion mapping described in Section A.3.

Figures A-3(a) and A-3(b) illustrate ratios of 0.2-second and 1.0-second response spectral accelerations at various return periods to 0.2-second and 1.0-second spectral accelerations at 108-year return period (corresponding to 50% PE in 75 years) for selected cities in California, WUS outside California, and CEUS based on new USGS mapping. Deterministic bounds on ground motions for long return periods have been incorporated where applicable in the curves in Figures A-3(a) and A-3(b). The curves indicate that ratios of MCE to expected earthquake ground motions in highly seismically active regions of California are typically equal to or less than 3 but typically exceed 4 to 5 in other parts of the WUS and 7 to 10 in the CEUS. As shown, in some locales of low seismicity and environments of characteristic large magnitude earthquakes having moderately long recurrence intervals, MCE-to-expected earthquake spectral ratios may exceed 10 to 20.

The decision to incorporate explicit design checks for this lower level design event was to get some parity between wind, flood and earthquake loads. The AASHTO LRFD provisions require essentially elastic design for the 100 year flood and the 100 mph wind which in many parts of the country is close to a 100 year wind load. Although the 50% PE in 75 year earthquake (108 year return period) only controls column design in parts of the western United States this recommendation provides for the first time some consistency in the expected performance of 100 year return period design events. The significant difference in the magnitude of earthquake loads with longer return periods is another reason why seismic design must consider much longer return period events. Both wind and flood loads tend to asymptotic values as the return period increases and in fact the ratio of a 2000 year/ 50 year wind load is in the range of 1.7 to 2.1 (Whalen and Simin 1998).

A.6 IMPACT STUDIES

Current AASHTO design uses a 500-year return period for defining the design earthquake. A more meaningful way to express this earthquake is in terms of probability of exceedance. A 500-year earthquake is one for which there is a 15% chance of exceedance in the 75 year life of the bridge. In other words there is a 15% chance that an earthquake will occur in the life of the bridge, which will be larger than the design earthquake. Whether this risk is acceptable or not depends on the probability of occurrence of the event, the consequences of the larger event, and the cost of reducing the consequences. A 15% PE in 75 years is by most standards a high chance of exceeding the design load. But to know if we should act to reduce the probability of exceedance we need to know the consequences of exceedance. To answer this question we need to know two things: (1) by how much will the design earthquake be exceeded and (2) the reserve capacity in the bridge due to conservative design provisions.

Most bridges have at least some capacity in reserve for extreme events. The present AASHTO Specification uses low R-Factors, a spectral shape based on $1/T^{2/3}$, generous seat widths, uncracked sections for analysis, low ϕ factors, Mononabe-Okabe coefficients for abutment wall design and the like. These criteria are based on engineering judgment and provide a measure of protection against large but infrequent earthquakes. But the degree of protection is unknown and the consequences of the larger events are uncertain and may be considerable. If the actual event is only 20% larger than the design event, damage will be slight, the consequences tolerable and the risk acceptable. On the other hand if the actual event is 200 to 400% larger, the reserve capacity may be exceeded, and damage and loss of access will likely be extensive. Here the risk may be unacceptable. If we use the 0.2-second and 1.0 second value of the spectral acceleration shown in Figures A-2(a) and A-2(b) as a measure of earthquake size , actual forces may exceed the design 500-year forces by factors that range from 1.5 (in Los Angeles) to 4.5 (in Charleston). Figures A-2(a) and A-2(b) illustrate this range for a number of cities in the US. Similar ratios to the 108 year forces are shown in Figures A-3(a) and A-3(b) and are approximately 3 for Los Angeles and exceeding 20 for Charleston. Reserve capacities as high as 4.5 are not explicitly embodied in the current AASHTO Specification and no assurance should be given regarding damage and access in these situations.

With this as background there were two options for the development of these new seismic design provisions. Either design explicitly for a larger event (3% PE in 75 year) but refine the provisions to reduce the conservatism and thus keep the costs about the same as the current provisions. Under this scenario, the degree of protection against larger earthquakes is quantified and based on scientific principles and engineering experience. Or design for a moderate sized event (15% in 75 year), and maintain the current conservative provisions as a measure of protection against larger events. In this scenario, the degree of protection is unknown and depends on intuition and engineering judgment. The project team selected the first option and as part of the development of the provisions performed a series of parameter studies to assess the cost impact of designing for the higher level event. These studies are summarized in Appendix G. In brief, they show that the net effect on the cost of a column and spread footing system is on the average 2% less than the current Division I-A provisions for multi-column bents and 16% less than Division I-A provisions for single column bents. These cost comparisons are based on the use of the more refined method for calculating overstrength factors and 2400 different column configurations including the seismic input of five different cities.

Another cost concern that arose during the development of the provisions was the impact of the longer return period on liquefaction. Two detailed case studies were performed using the new and existing provisions and these are summarized in Appendix H. These examples demonstrated that application of the new provisions, with the inclusion of inelastic deformation in the piles as a result of lateral flow, would not be significantly more costly than the application of the current provisions. Hence the objective of having a quantifiable degree of protection against larger earthquakes for similar costs was achieved.

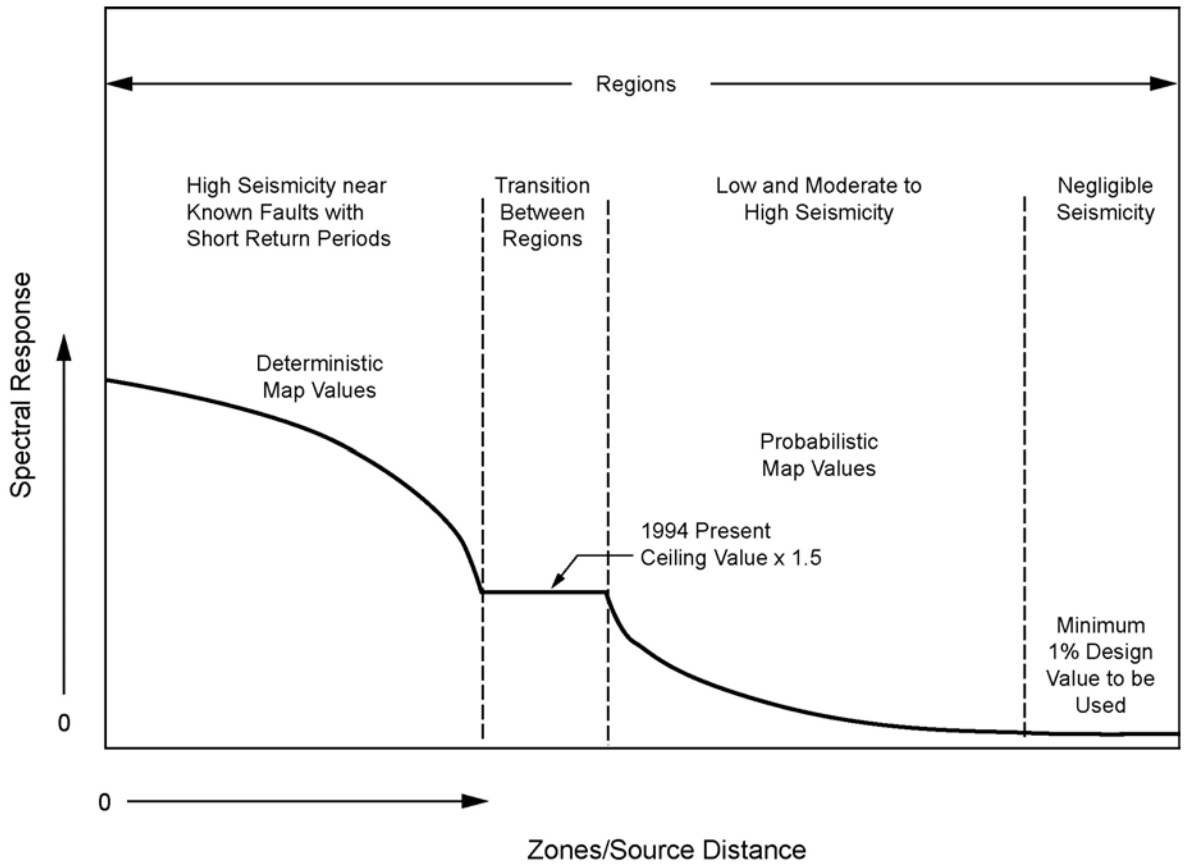

Figure A-1 Procedure for incorporation of deterministic bounds in the maximum considered earthquake (MCE) ground motion map of the 1997 NEHRP Provisions (BSSC, 1998).

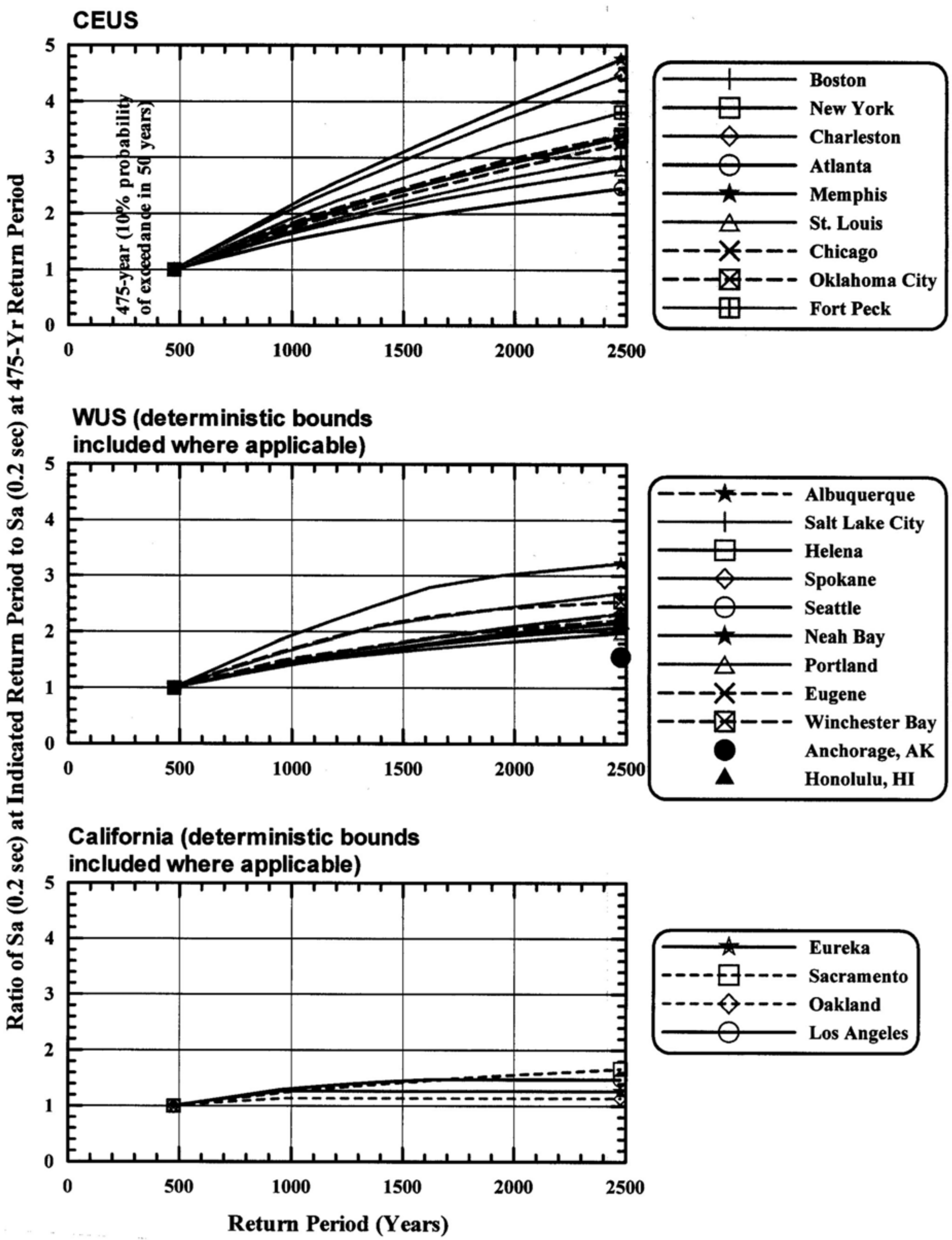

Figure A-2(a) Ratios of 0.2 second spectral acceleration at return period to 0.2-second spectral acceleration at 475-year return period

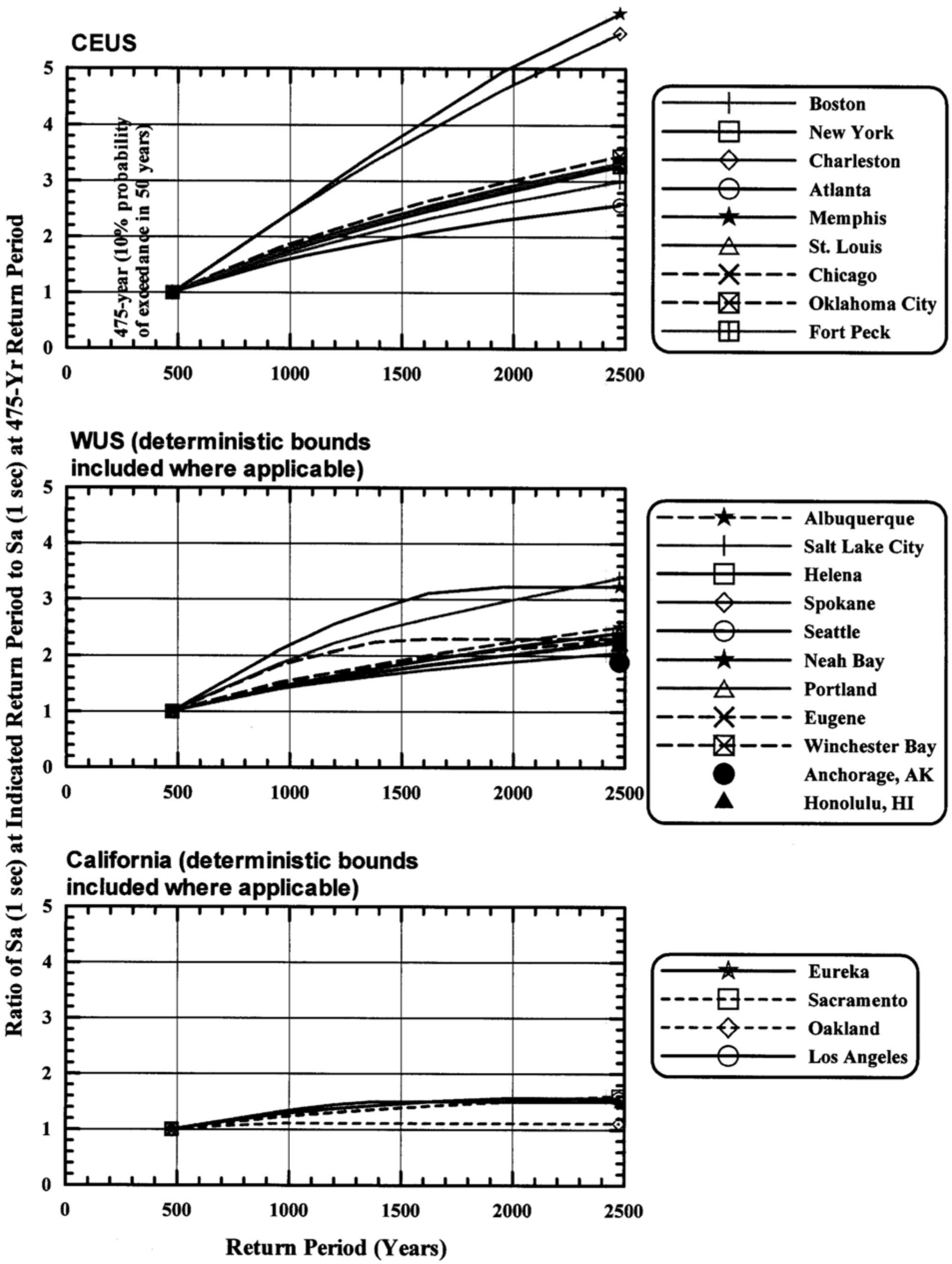

Figure A-2(b) Ratios of 1.0 second spectral acceleration at return period to 1.0-second spectral acceleration at 475-year return period

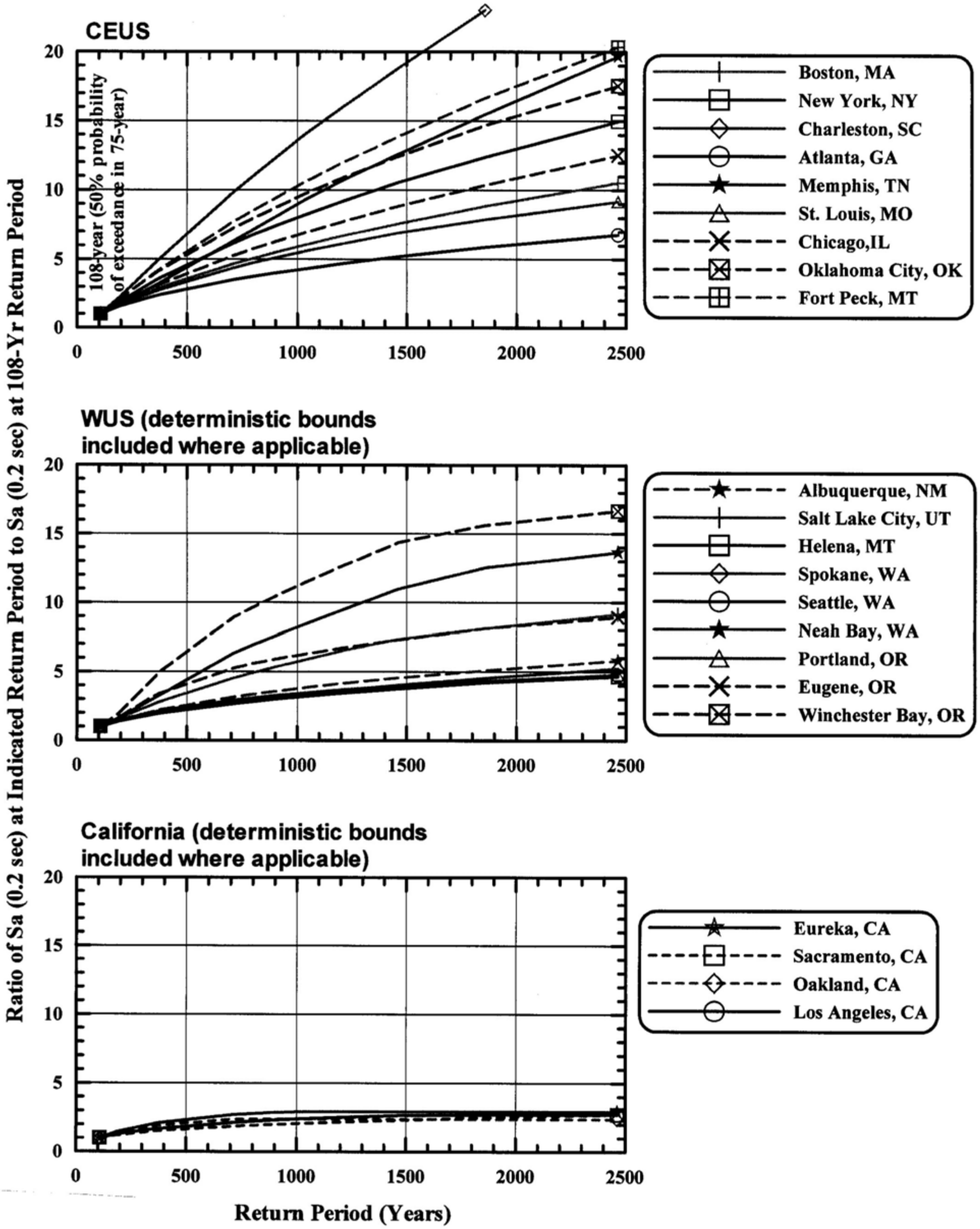

Figure A-3(a) Ratios of 0.2-second spectral acceleration at return period to 0.2-second spectral acceleration at 108-year return period

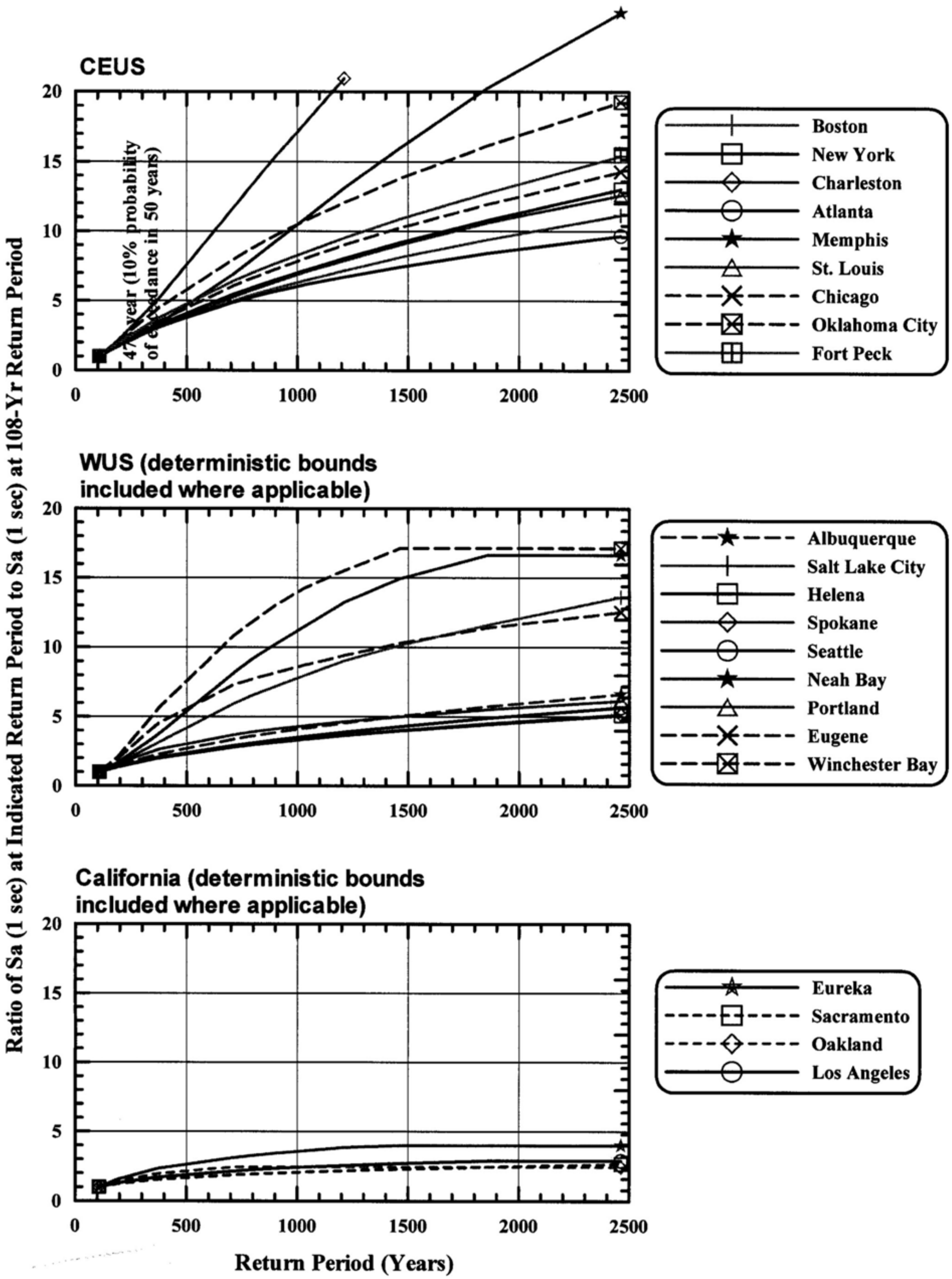

Figure A-3(b) Ratios of 1.0-second spectral acceleration at return period to 1.0-second spectral acceleration at 108-year return period